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ANCHORED BULKHEADS

by Karl Terzaghi, Hon. M. ASCE

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ANCHORED BULKHEADS

BY KARL TERZAGHI,[1] HON. M. ASCE

[1] Prof. of the Practice of Civ. Eng., Harvard Univ., Cambridge, Mass.

SYNOPSIS

In Part I of this paper the fundamental assumptions made in existing methods of bulkhead design are compared with observational data published in the past few decades. The investigation shows that the discrepancies are too important to be ignored in the design.

Part II deals with the evaluation of the forces acting on anchored bulkheads and with the safety requirements. The importance of the errors involved in the estimates of the bending moments and the soil reactions depend, to a large extent, on the type of soils involved in the design problem, on the degree of complexity of the structure of the soil strata, on the degree of uniformity of the backfill material, and on the time and labor invested in the subsoil exploration. Therefore, it would be economically unjustified to select the allowable stresses and safety factors in the construction materials without taking into consideration the degree of reliability of the available data. The selection of these values requires experience and good judgment.

If the designer has only the results of a conventional subsoil exploration available, the problem must be solved on the basis of empirical values such as those contained in this paper. In connection with the design of bulkheads, involving unusual features not treated in this paper, the designer must improvise the procedure; and, in doing so, it will be found advantageous to consult the results of the experimental investigations summarized in Part I.

Notation.—The letter symbols in this paper are defined where they first appear, in the text or by illustration, and are assembled alphabetically, for convenience of reference, in the Appendix.

I. THEORETICAL AND REAL PRESSURES ON ANCHORED BULKHEADS

Types of Bulkheads

Bulkheads are the retaining walls of the waterfront. Depending on the depth of penetration of the sheet piles, they are commonly divided into bulkheads with free earth support (Fig. 1(*a*)) and bulkheads with fixed earth support (Fig. 1(*b*)). The sheet piles of the second type are driven so deep into the ground that the bulkhead can fail only by bending or because of inadequate anchorage. If the sheet piles have been driven their full length (or almost full length) into natural ground, and the natural ground has then been removed by excavation, the bulkhead is referred to as a "dredge bulkhead." On the other hand, if the surface of the natural ground is approximately at the level of the dredge line, and the material between the natural ground surface and the level of the upper edge of the sheet piles was deposited after the sheet piles were driven, the bulkhead is called a "fill bulkhead."

Classical Design Assumptions

Although anchored bulkheads were probably in use in pre-Roman times, it appears that no attempt was made to design them on the basis of earth pressure computations until about 1910, when H. Krey, in Berlin, Germany, began to investigate the problem. Somewhat later he published an analytical procedure for bulkhead design. During the subsequent decades the Krey method was supplemented by various refinements including the "elastic line" and the "equivalent beam" method. In the United States, methods for the design of bulkheads on the basis of the classical earth pressure theories were developed and introduced into the practice of structural design by A. P. Pennoyer.[2] The fundamental assumptions on which all these procedures are based are illustrated in Fig. 2.

[2] "Design of Sheet-Piling Bulkheads," by Raymond A. Pennoyer, *Civil Engineering*, November, 1933, p. 615.

Fig. 2(*a*) represents a bulkhead with free earth support. The inner face of the bulkhead is assumed to be acted upon by the active Coulomb pressure (pressure triangle abc). The force that resists the outward movement of the buried part eb is represented by the shaded triangle bef.

Fig. 2(*b*) shows a bulkhead with fixed earth support. The triangle abc represents the active earth pressure on the inner face. The assumed resistance against the outward movement of the buried part of the bulkhead is indicated by the pressure area edb. Since the elastic line of the sheet piles forming an anchored bulkhead with fixed earth support is assumed to pass below a certain point b onto the right-hand, or inner, side of the original position of the bulkhead, it is also assumed that the earth pressure on the inner face changes at the elevation of point b from active to passive (line cc_1, Fig. 2(*b*)), whereas the pressure on the outer face changes at the same point from passive to active (line gg_1). The depth of penetration of the sheet piles (D) is computed in such a manner that the elastic line of the sheet piles satisfies the condition of fixed earth support. The computation is performed either by trial and error

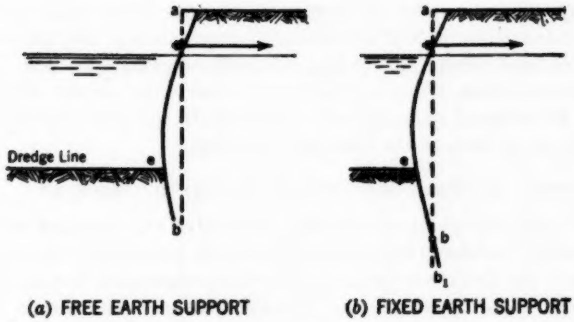


Fig. 1.—The Two Extreme Types of Earth Support

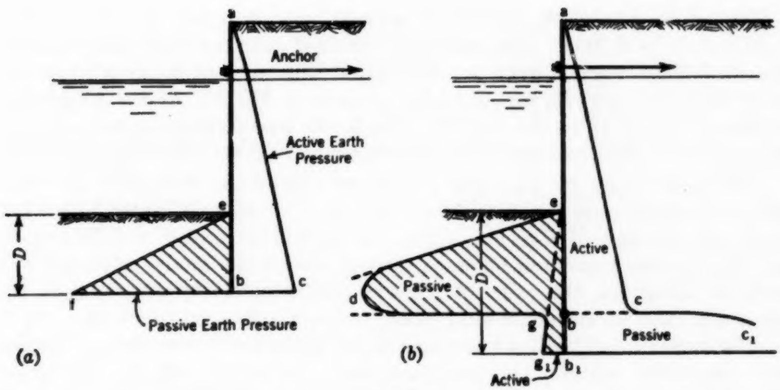


Fig. 2.—Classical Conceptions Concerning Distribution of Earth Pressure on Bulkheads with (a) Free and (b) Fixed Earth Support

or on the basis of supplementary simplifying assumptions.

At the time when the assumptions illustrated by Fig. 2 were originated, the physical conditions for the validity of these assumptions were still unknown. Since that time, a large number of observational data have been accumulated which are incompatible with the original assumptions. Nevertheless, the methods of bulkhead design remained practically unchanged. The following sections of Part I contain a summary of those observational data which are at variance with the original assumptions. These data will then be used in Part II as a basis for a revision of the methods of design.

Effect of Wall Movements on Earth Pressure

The design methods illustrated in Fig. 2 involve the assumption that an infinitesimal yield of a lateral support is sufficient to reduce the intensity of the earth pressure to its minimum value and that a further yield has no influence on this pressure. This assumption is incompatible with the results of large-scale earth pressure tests[3] which furnished accurate information concerning

[3] "Large Retaining-Wall Tests. I. Pressure of Dry Sands," by Karl Terzaghi, *Engineering News-Record*, Vol. 112, 1934, pp. 136–140.

relations among (1) the yield ratio $d = Y/H$ of a vertical wall (Fig. 3(*b*)) with height $H = 5$ ft, backfilled with coarse, clean sand in a loose or in a compacted state; (2) the coefficient of active earth pressure K_A; (3) the mobilized part ϕ' of the angle of internal friction ϕ; and (4) the angle of wall friction δ. The tests led to the following conclusions:

The assumed value K_A for the dense sand was its minimum value corresponding to $d = 0.0005$. This value was retained until the yield ratio became equal to 0.002. Further yield was associated with an increase of K_A toward the minimum value of K_A for loose sand as shown in Fig. 3. At $d = 0.0046$ an audible slip occurred in the backfill. Along the line of intersection between the surface of sliding and the surface of the backfill a low fault scarp appeared.

The value of K_A for loose sand decreased from 0.4 to 0.30 while the yield ratio d increased from zero to 0.0003. Further yield was associated with a less important decrease of K_A (Fig. 3(*a*)). At a yield ratio of $d = 0.007$, corresponding to the maximum distance through which the model retaining wall could be advanced, the value of K_A was still considerably greater than the minimum value for the dense sand (0.23) and no slip had yet occurred.

The angle of wall friction δ assumed its full value before the internal friction was completely active. On the other hand, when the wall was advanced toward the backfill over a distance of 0.002 H, the wall friction was still much smaller than its maximum value.

At any stage of the tests, as soon as the wall ceased to yield, both the angle of internal friction ϕ' and the angle of wall friction δ decreased slightly at a decreasing rate. Part of the decrease was probably caused by the fact that the backfill of the model retaining wall was subject to intermittent vibrations caused by passing trains.

The backfill of fill bulkheads is almost never compacted by artificial methods, and the average yield of the bulkhead hardly exceeds a fraction of 1% the height of the bulkhead. Therefore, it is unlikely that the lateral pressure of a sand backfill on an anchored bulkhead is as low as the active earth pressure of the fill material.

In the tests illustrated in Fig.3, the wall was not allowed to yield until the

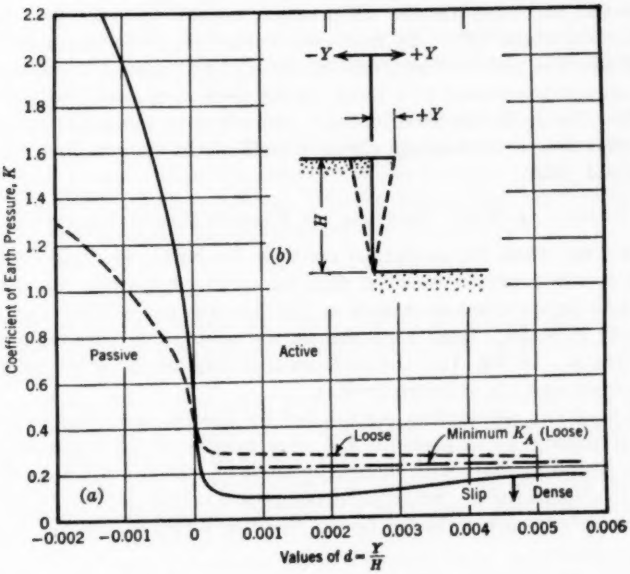


FIG. 3.—INFLUENCE OF RELATIVE DENSITY OF SAND AND EFFECTIVE YIELD Y ON COEFFICIENT OF EARTH PRESSURE

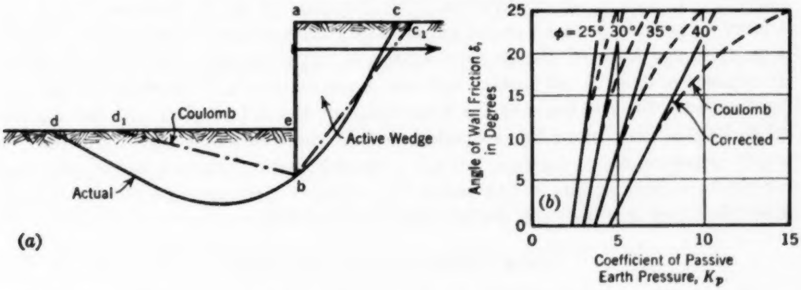
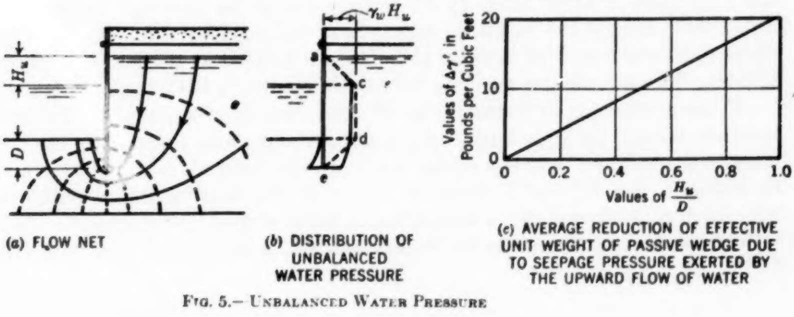


FIG. 4.—DIAGRAMS ILLUSTRATING ERRORS INVOLVED IN COULOMB'S ASSUMPTION THAT THE PASSIVE FAILURE TAKES PLACE ALONG A PLANE SURFACE OF SLIDING



(a) FLOW NET

(b) DISTRIBUTION OF UNBALANCED WATER PRESSURE

(c) AVERAGE REDUCTION OF EFFECTIVE UNIT WEIGHT OF PASSIVE WEDGE DUE TO SEEPAGE PRESSURE EXERTED BY THE UPWARD FLOW OF WATER

FIG. 5.—UNBALANCED WATER PRESSURE

entire backfill had been placed. In practice, backfilling operations and yield take place simultaneously. In this case distinction must be made between total and effective yield. The term "effective yield" refers to that part of the total horizontal movement of a point on the back of a lateral support which occurs after the point has been buried. The value of the coefficient of earth pressure depends on the average effective yield of the support and not on the average total yield.

Effect of Wall Friction on Passive Earth Pressure

At the time when the analytical methods for bulkhead design came into existence, it was generally believed that the Coulomb method for computing passive earth pressure was as reliable as the Coulomb procedure for computing active earth pressure. Both are based on the assumption that the surface of sliding is plane. In Fig. 4(*a*) the surfaces of sliding are indicated by lines bc_1 (active wedge) and bd_1 (passive wedge).

If the Coulomb assumption represented by line bd_1 were justified, the coefficient of passive earth pressure K_P, of a sand with an angle of internal friction ϕ, would increase with increasing angle of wall friction δ, as shown in Fig. 4(*b*) by dashed lines. Subsequent theoretical investigations[4,5] have shown that the base of the active wedge (line bc, Fig. 4(*a*)) really is almost a plane. Therefore, the Coulomb minimum value for the active earth pressure is almost correct. However, in connection with the passive earth pressure, both the theory of plasticity and various experimental investigations led to the conclusion that the base of the passive wedge is not even approximately a plane. It consists of one curved section and one plane section, as indicated in Fig. 4(*a*) by line bd. On the basis of the knowledge of the actual shape of the surface of sliding, it was found by computation that the coefficient K_P of the passive earth pressure on a vertical lateral (support (ratio between horizontal and vertical pressure at any depth below the surface) increases in accordance with the solid lines, and not the dashed lines, in Fig. 4(*b*).

[4] "Theoretical Soil Mechanics," by Karl Terzaghi, John Wiley & Sons, Inc., New York, N. Y., 1943, pp. 42–53.
[5] *Ibid.*, pp. 100–105.

Unbalanced Water Pressure

If an anchored bulkhead is located at the seashore, the earth pressure on the inner face of the sheet piles is a maximum at low tide. At the same time, the inner face is acted upon by an unbalanced water pressure because the water table behind the bulkhead lags behind the receding tide. Unbalanced water pressures may also develop at bulkheads located at the shores of rivers or lakes, during a rapidly receding high water or during heavy rainstorms.

If the coefficients of permeability of the strata in contact with the bulkhead are known, the distribution of the unbalanced water pressure on the two faces of the bulkhead corresponding to a hydraulic head H_u can be determined by means of the flow net method, as shown in Fig. 5 for a dredge bulkhead with sheet piles driven into a homogeneous mass of fine, uniform sand.[6] Fig.

[6] "Theoretical Soil Mechanics," by Karl Terzaghi, John Wiley & Sons, Inc., New York, N. Y., 1943, p. 252.

5(a) represents the flow net and Fig. 5(b), the corresponding distribution of the unbalanced part of the water pressure over the two faces of the bulkhead. Accurate flow nets for anchored bulkheads in contact with homogeneous sand have been published by J. McNamee.[7]

[7] "Seepage into a Sheeted Excavation," by J. McNamee, *Geotechnique*, Vol. I, 1949, pp. 229-241.

If the permeability of all the soil strata in contact with the bulkhead is practically the same, it can be assumed, without serious error, that the inner face of the bulkhead at any depth between the dredge line and the outside water level is acted upon by an unbalanced water pressure,

$$P_u = \gamma_w H_u \qquad (1)$$

in which γ_w is the unit weight of water. Below the dredge line, P_u decreases from $\gamma_w H_u$ to zero at the lower edge of the sheet piles, as indicated in Fig. 5(b) by the straight line de.

If the permeability varies in vertical directions between wide limits, the determination of the distribution of the unbalanced water pressure may require the construction of a flow net.

When the water table in the backfill is above the free water level, the water percolates through the backfill in a downward direction, flows around the lower edge of the sheet piles, and rises beyond the outer face, as indicated in Fig. 5(a). The seepage pressure exerted by the rising ground water reduces the effective unit weight of the soil in contact with the outer face of the bulkhead and, as a consequence, it reduces the passive earth pressure. If i is the average hydraulic gradient in the soil adjoining the outer face, the corresponding reduction of the submerged unit weight γ' of the soil is[8]

[8] "Soil Mechanics in Engineering Practice," by Karl Terzaghi and R. B. Peck, John Wiley & Sons, Inc., New York, N. Y., 1948, p. 54.

$$\Delta\gamma' = i\,\gamma_w \qquad (2)$$

Under the conditions illustrated in Fig. 5(a), the average value of i is somewhat smaller than $\frac{H_u}{3\,D}$. Hence, the effective unit weight of the soil in contact with the outer face of the bulkhead will be slightly greater than

$$\gamma' - \Delta\gamma' = \gamma' - \gamma_u \frac{H_u}{3\,D} \qquad (3)$$

The relationship between $\Delta\gamma'$ and H_u/D is shown in Fig. 5(c). Within the backfill, the water percolates in a downward direction and it produces an increase of the effective unit weight of the fill by $\Delta\gamma''$. However $\Delta\gamma''$ is very much smaller than $\Delta\gamma'$ and does not require consideration.

Lateral Pressure Resulting from Line Loads

The first attempts to compute the lateral earth pressure p' per unit of length of a wall due to line loads, q', per unit of length, were made in the Nineteenth Century, before experimental data were available. Based on the

Coulomb earth pressure theory, they led to the conclusion[9] that the intensity

[9] "Soil Mechanics in Engineering Practice," by Karl Terzaghi and R. B. Peck, John Wiley & Sons, Inc., New York, N. Y., 1948, p. 160.

and the position of the center of the pressure caused by the line load depends on the angle of internal friction ϕ and the angle of wall friction δ. These conclusions are illustrated in Figs. 6(*b*) and 6(*a*). In Fig. 6(*b*) the abscissa of point s_1 represents the ratio m_1 between the width of the top surface of the "sliding wedge" (line ac, Fig. 4(*a*)) and the height H of a wall with a smooth back ($\delta = 0$) acted upon by the lateral pressure of very loose sand ($\phi = 30°$). The abscissa of point s_2 represents the corresponding m_1-value for a wall with a rough back ($\delta = 25°$), backfilled with dense sand ($\phi = 40°$). The ordinates of the dash-double-dot curves that pass through these points represent the ratio between the lateral pressure p' and the intensity q' of the line load computed by means of the Coulomb theory.

At any given value of ϕ and δ the width $m_1 H$ of the top of the sliding wedge decreases slightly with increasing values of q'. If the line load is applied at a distance of less than $m_1 H$ from the crest of the wall, the value of p'/q' is independent of the position of the line load with reference to the crest of the wall. Therefore, on the left-hand side of points s_1 and s_2 the lines representing the $\frac{p'}{q'}$ -values are horizontal. If the line load moves to the right-hand side of points s_1 (loose sand) or s_2 (dense sand), the lateral pressure decreases rapidly and finally equals zero. The Coulomb theory also leads to the conclusion that the lateral pressure caused by a line load acts only on a narrow, horizontal strip. The elevation of this strip above the foot of the wall depends on the values ϕ and δ, and on the value m, which determines the position of the line load q' with reference to the crest of the wall (Fig. 6(*c*)). In Fig. 6(*a*) the position of the strips acted upon by the lateral force p' is indicated by horizontal arrows. To each arrow are added the values of ϕ and δ on which the computation of its position was based. In each of the four diagrams in Fig. 6(*a*) the two arrows indicate the extreme positions between which the theoretical center of the pressure caused by line loads on sand fills with different density may be located.

The conclusions based on the Coulomb theory concerning the lateral pressure resulting from line loads remained practically unchallenged until both the intensity and distribution of the lateral pressure were determined experimentally by E. Gerber[10] and M. G. Spangler,[11] M. ASCE. The backfill used

[10] "Untersuchungen über die Druckverteilung im örtlich belasteten Sand," by E. Gerber, Zürich, Switzerland, 1929.

[11] "Horizontal Pressures on Retaining Walls Due to Concentrated Surface Loads," by M. G. Spangler, *Bulletin No. 140*, Iowa Eng. Experiment Station, Iowa City, Iowa.

by Mr. Gerber consisted of clean, uniform river sand with a grain size between 0.2 mm and 1.5 mm. The lateral support was practically rigid. It consisted of the concrete side wall of a rectangular pit, with a depth of 31 in. The lateral pressures caused by the line load were measured by means of pressure cells arranged in vertical rows. Mr. Spangler used as a backfill material pit run gravel with 13% particles passing the 200-mesh sieve. The lateral support consisted of a reinforced concrete cantilever wall, 84 in. high and 6 in. thick, which was free to tilt about the outer edge of the base of the base plate. In spite of the

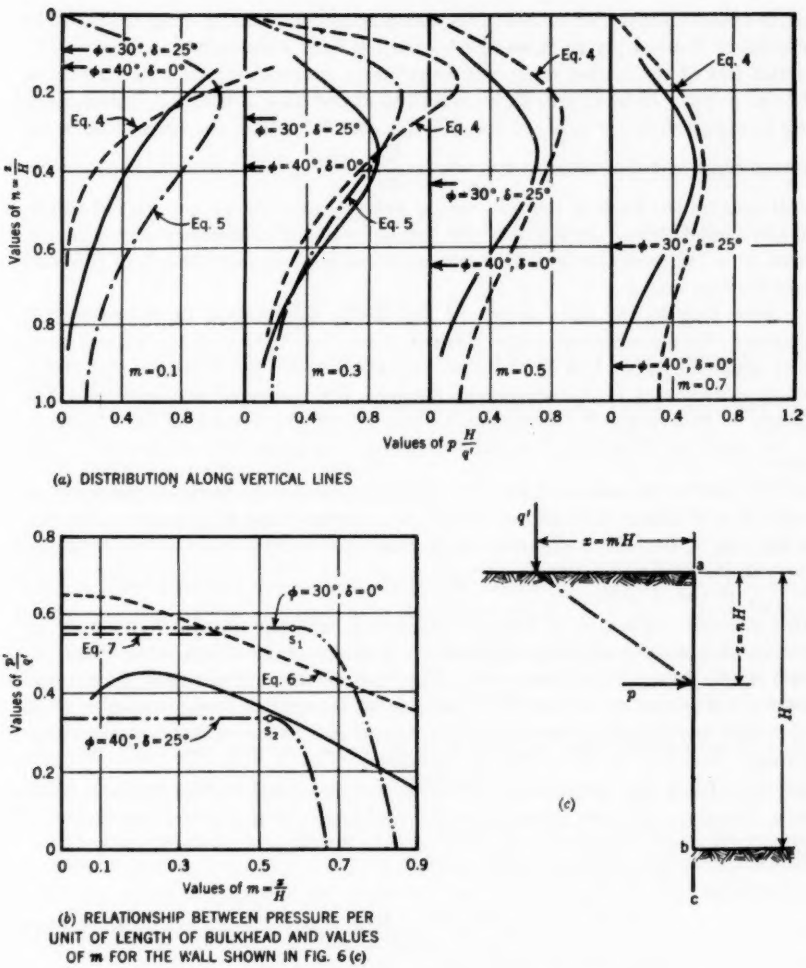


FIG. 6.—MEASURED (PLAIN CURVES) AND COMPUTED (DASH AND DASH-DOT CURVES) LATERAL PRESSURE DUE TO LINE LOADS

differences in the test conditions in these two sets of tests, there are no essential differences between the test results.

The solid curves in Fig. 6(*a*) represent the results of one of Mr. Gerber's series of tests performed on backfills with a height $H = 32$ in. (Fig. 6(*c*)). The surcharge, 0.4 ton per sq ft, covered a strip 6.3 in. wide and 25 in. long. The center line of the loaded strip was established, successively, at a distance $x =$ 3.1 in., 9.4 in., 15.5 in., and 22 in. from the upper edge of the wall, corresponding to values of m ($= x/H$) of 0.1, 0.3, 0.5, and 0.7. The abscissas of the solid curves represent the values of the ratio $p \frac{H}{q'}$, p being the horizontal pressure per unit area on the back of the wall, and q' being the surcharge, per unit of length of the loaded strip. In Fig. 6(*b*) the ordinates of the solid curve represent the ratio p'/q' between the measured lateral pressure p' per unit length of the wall and the line load q'.

According to the data shown in Fig. 6 the information furnished by the Coulomb theory concerning the intensity and distribution of the lateral pressure resulting from line loads is incompatible with the experimental data. More satisfactory is the agreement between the measured pressures and the theory of Boussinesq.[12] According to this theory the horizontal unit pressure

[12] "Theoretical Soil Mechanics," by Karl Terzaghi, John Wiley & Sons, Inc., New York, N. Y., 1943, p. 376.

σ_z on a vertical section ac in Fig. 6(*c*) through a semi-infinite elastic medium, at a depth $n\,H$ below the surface, caused by a surface load q' per unit of length, acting on a line at a distance $m\,H$ from the vertical section is equal to $\sigma_z = \frac{2}{\pi}\frac{q'}{H}\frac{m^2\,n}{(m^2 + n^2)^2}$. However, the application of the line load tends to produce a lateral deflection of the vertical section, and the flexural rigidity of the bulkhead interferes with that deflection. In order to obtain the lateral pressure on a relatively rigid diaphragm ab in Fig. 6(*c*), located at the site of the vertical section,[13] a second and equal line load q' must be applied at a distance $m\,H$ on

[13] "Pressure Distribution on Retaining Walls," *Proceedings*, First International Conference on Soil Mechanics and Foundation Eng., Discussion by R. D. Mindlin, Cambridge, Mass., 1936, Vol. 3, pp. 155–156.

the right-hand side of point a. This second line load doubles the unit pressure; therefore, the unit pressure on the wall at depth $n\,H$ below the surface is $p = 2\,\sigma_z = \frac{4}{\pi}\frac{q'}{H}\frac{m^2\,n}{(m^2 + n^2)^2}$; and

$$p\,\frac{H}{q'} = \frac{4}{\pi}\,\frac{m^2\,n}{(m^2 + n^2)^2} \quad \text{.....................} (4)$$

Eq. 4 is represented in Fig. 6(*a*) by dash-line curves. For values of m greater than about 0.4 the agreement between theory and observation is fair. However, for values smaller than 0.4, the discrepancy between observed and computed values increases with decreasing values of m (as shown by dash-line curves in Fig. 6(*a*) for $m = 0.1$ and $m = 0.3$). For such values of m (less than 0.4) it was found, by trial and error, that the observed pressure distribution has greater similarity to the computed distribution for $m = 0.4$ which is determined by the equation:

$$p \frac{H}{q'} = \frac{0.203\, n}{(0.16 + n^2)^2} \quad \text{(5)}$$

Eq. 5 is represented in Fig. 6(*a*) by dash-dot curves.

For values of m greater than 0.4 the lateral pressure p' per unit of length of the wall is $p' = \int_{n=0}^{n=1} p\, H\, dn = \frac{2}{\pi} \frac{q'}{m^2 + 1}$; from which

$$\frac{p'}{q'} = \frac{2}{\pi\,(m^2 + 1)} \quad \text{(6)}$$

For values of m smaller than 0.4 Eq. 6 must be replaced, in accordance with Eq. 5, by

$$\frac{p'}{q'} = \frac{2}{\pi\,(0.16 + 1)} = 0.55 \quad \text{(7)}$$

In Fig. 6(*b*), Eq. 6 is represented by a dash line and Eq. 7 is represented by a dash-dot line.

The diagrams in Figs. 6(*a*) and 6(*b*) show that the values obtained by use of Eqs. 4 to 7 are consistently greater than the measured values. Part of the difference is due to the fact that the Gerber tests[10] were made with line loads having a length of not more than 0.8 H, whereas the computed values refer to line loads with infinite length. The remainder of the difference results from the fact that the Boussinesq theory is strictly applicable only to perfectly elastic materials. Since the computed values are too high, the differences partly compensate for the fact that the distribution of the $\frac{p'}{q'}$-values, under field conditions, may deviate to some extent from the observational curves which have been constructed on the basis of the test results reported by Mr. Gerber. The deviations are chiefly caused by the complex stress-strain relations for sands. Because of these relations the pressure distribution depends not only on the values of m and n contained in Eqs. 4 and 5, but also, to some extent, on the value of the ratio q'/H. Mr. Gerber's data were obtained only for one value of this ratio.

The observational data shown in Fig. 6 eliminated the Coulomb theory as a source of information concerning the lateral pressure produced by line loads, and they made it possible to establish empirical equations for estimating upper limiting values of the pressures produced by such loads.

Lateral Pressure Resulting from Point Loads

Intensity and distribution of the lateral pressure resulting from point loads were investigated by Messrs. Gerber[10] and Spangler.[11] The test results were practically identical. The point load in Mr. Gerber's tests consisted of a loaded circular slab with a diameter of 14 in. The slab rested upon the horizontal surface of a layer of clean, coarse sand with a depth of 32 in., at varying distances from the crest of the wall. Fig. 7 shows the distribution of the lateral pressure over the back of the wall. The pressure is greatest along the line of intersection ab between the wall and a vertical plane through the center of the

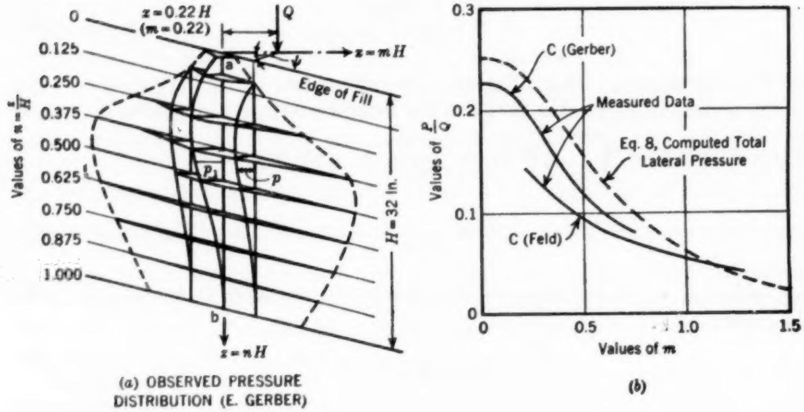


FIG. 7.—LATERAL PRESSURE DUE TO POINT LOADS

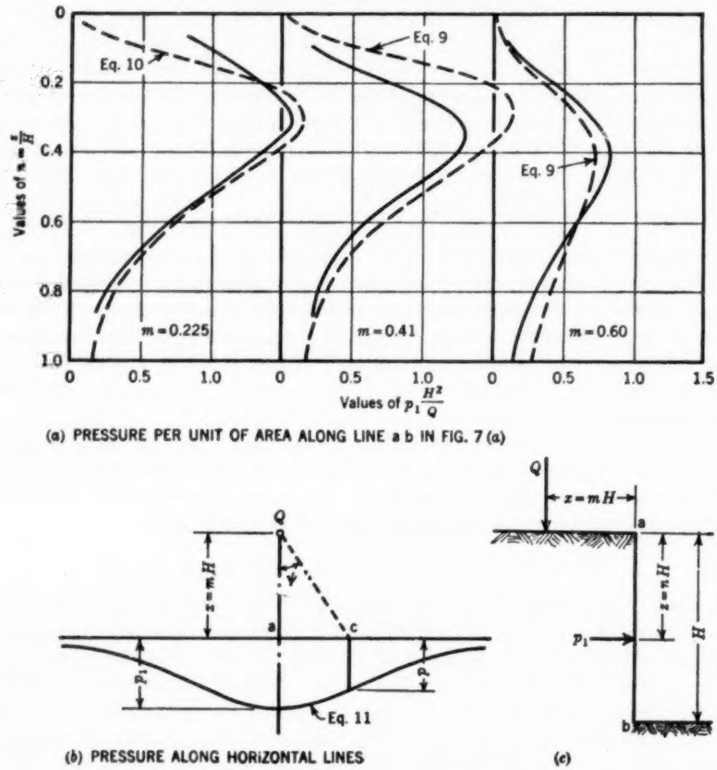


FIG. 8.—LATERAL PRESSURE PRODUCED BY POINT LOADS ON LATERAL SUPPORT

load at right angles to the wall. Along this line, the unit pressure p_1 first increases with increasing depth, assumes a maximum value at a depth which is somewhat greater than the distance between load and wall, and then decreases again. At any depth the pressure decreases in horizontal directions with increasing distance from line ab.

Fig. 7(*b*) shows the relationship between $m = x/H$ and the total intensit P of the lateral pressure caused by the point load Q. Curve C(Gerber) is based on test results[10] and curve C(Feld) on test data published by Jacob Feld,[14] M. ASCE. The lack of perfect agreement between observational curves obtained by different investigators is chiefly a result of the fact that the values of P/Q depend not only on m but also on various other factors such as the deflection ratio d for the lateral support and the ratio between Q and the ultimate bearing capacity of the backfill. Upper limiting values for P can be obtained by use of the empirical equation:

$$\frac{P}{Q} = \frac{0.25}{(1 + m^2)^2} \qquad (8)$$

which is based on Mr. Gerber's test results. In Fig. 7(*b*) Eq. 8 is represented by a dashed curve.

None of the existing theories (1953) accounts satisfactorily for the distribution over the inner face of a wall of the lateral pressure produced by a point load Q. For values of m greater than 0.4 the unit pressures p_1 along line ab, Fig. 7(*a*), can be estimated roughly by use of the empirical equation:

$$p_1 \frac{H^2}{Q} = 1.77 \frac{m^2 n^2}{(m^2 + n^2)^3} \qquad (9)$$

For values of m less than 0.4, a better approximation is obtained by assigning to m in Eq. 9 a constant value $m = 0.4$, thus—

$$p_1 \frac{H^2}{Q} = \frac{0.28\, n^2}{(0.16 + n^2)^3} \qquad (10)$$

In Fig. 8(*a*) the solid curves represent the Gerber test results, and the dashed curves represent Eqs. 9 and 10.

The intensity of the lateral pressure on the back of the wall on both sides of the line ab in Fig. 7(*a*) is a complicated function of the depth below the crest of the wall and the horizontal distance from the line ab. If the point load Q is located at a distance $m\,H$ from the wall, an upper limiting value for the unit pressure p at a depth $n\,H$ below the surface of the fill and at a horizontal distance $m\,H \tan \psi$ (Fig. 8(*b*)) from the vertical line through point a can be obtained by use of the empirical equation:

$$p = p_1 \cos^2 (1.1\, \psi) \qquad (11)$$

In Eq. 11, p_1 is the unit pressure on the wall at depth $n\,H$ for $\psi = 0$. The

[14] "Lateral Earth Pressure: The Accurate Experimental Determination of the Lateral Earth Pressure, Together with a Resume of Previous Experiments," by Jacob Feld, *Transactions*, ASCE, Vol. LXXXVI, 1923, pp. 1448–1505.

value of p_l is determined by Eq. 9 for m greater than 0.4 and by Eq. 10 for values of m smaller than 0.4. Eq. 11 is an empirical equation based on Mr. Gerber's test results.

Distribution of Earth Pressure

Theory and observation have shown that the distribution of the pressure on a lateral support is by no means necessarily in accordance with the Coulomb theory because it depends largely on the type of yield.[15] This fact is illustrated by Fig. 9 which represents the distribution of the lateral pressure on the back of a lateral support for three different types of yield. The effective yield at any depth below the surface is indicated by the width of the shaded area at that depth.

[15] "General Wedge Theory of Earth Pressure," by Karl Terzaghi, *Transactions*, ASCE, Vol. 106, 1941, pp. 68–80.

In connection with anchored bulkheads, the validity of the Coulomb theory was questioned for the first time in 1906 by Danish engineers on purely empirical grounds. It was argued that the lateral pressure on bulkheads is a minimum midway between the dredge line and the anchor line, as shown in Fig. 9(c). This conception received experimental support by tests performed by J.P.R.N. Stroyer.[16] This concept received considerable attention because a lateral pressure with the distribution shown in Fig. 9(c) produces very much smaller bending moments in the sheet piling than a pressure with Coulomb distribution (Fig. 9(a)) and equal total intensity. In 1938 J. Ohde computed the distribution of the earth pressure on flexible walls with fixed upper and lower edges and he also found that the pressure distribution should have the characteristics shown in Fig. 9(c). His theoretical conclusions were confirmed by large-scale tests performed by H. Press in 1948,[17] on a flexible wall 27 ft high. The inner face of the wall was paved with pressure cells.

[16] "Earth Pressure on Flexible Walls," by J. P. R. N. Stroyer, *Journal*, Inst. of C. E., Vol. I, 1935, p. 94.

[17] "Über die Druckverteilung im Boden hinter Wänden verschiedener Art," by H. Press, *Bautechnik Archiv.*, W. Ernst and Son, Berlin, Germany, 1948, pp. 36–54.

Between the years 1944 and 1948 G. P. Tschebotarioff, M. ASCE, performed large-scale tests on bulkhead models.[18] He measured the lateral deflection and the extreme fiber stresses in the bulkheads at different elevations above their lower edge and he computed the pressure distribution on the basis of these data. He obtained the pressure distribution illustrated by Fig. 9(c) for bulkheads of the dredge type only. The distribution of the earth pressure on the inner face of the models of fill bulkheads was found to be similar to that shown in Figs. 10 and 11. The tests represented by Fig. 10 were made with a relatively stiff bulkhead with a maximum deflection equal to about 0.1% of the vertical distance H between the anchor line and the dredge line. The corresponding K_A-value was nearly 0.4, which is approximately equal to the coefficient of earth pressure at rest. The intensity and the distribution of the active earth pressure on more flexible bulkheads, having a deflection ratio of

[18] "Final Report. Large Scale Earth Pressure Tests with Model Flexible Bulkheads," by G. P. Tschebotarioff, Princeton Univ., Princeton, N. J., 1949.

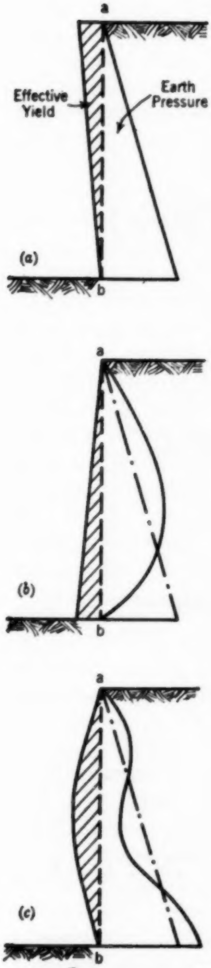


FIG. 9.—INFLUENCE OF MOVEMENT TYPE ON PRESSURE DISTRIBUTION

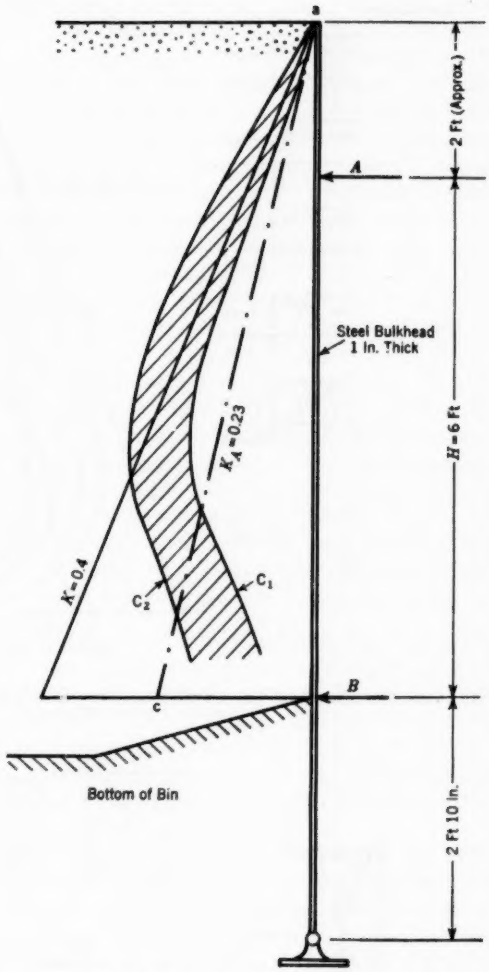


FIG. 10.—INTENSITY AND DISTRIBUTION OF ACTIVE EARTH-PRESSURE ON INNER FACE OF RELATIVELY STIFF MODEL BULKHEAD

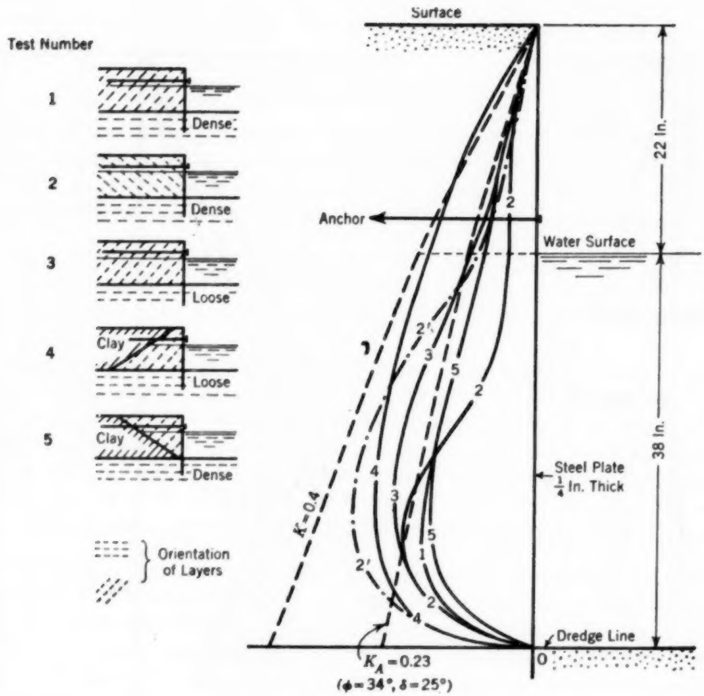


FIG. 11.—INTENSITY AND DISTRIBUTION OF ACTIVE EARTH PRESSURE ON THE INNER FACE OF RELATIVELY FLEXIBLE MODEL BULKHEADS

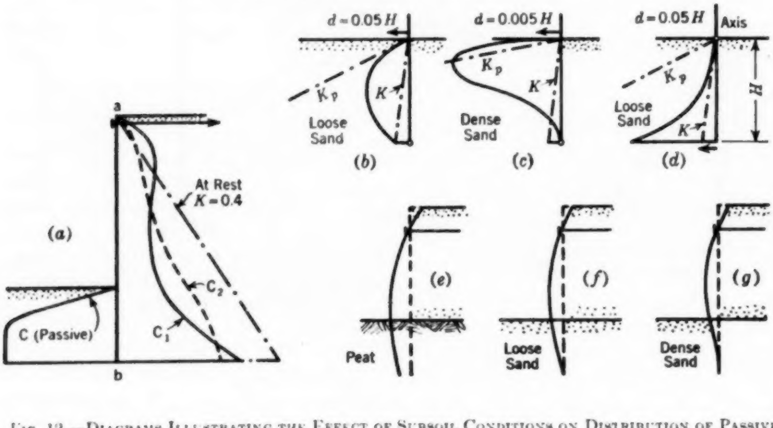


FIG. 12.—DIAGRAMS ILLUSTRATING THE EFFECT OF SUBSOIL CONDITIONS ON DISTRIBUTION OF PASSIVE EARTH PRESSURE AND ON TYPE OF BULKHEAD DEFLECTION

about 0.5%, is shown in Fig. 11. The corresponding K_A-value approximated that of the active earth pressure. Curve 2′ represents Test No. 2 after compaction of the fill by vibration. The line marked $K_A = 0.23$ represents the Coulomb pressure computed on the assumption that $\phi = 34°$ and $\delta = 25°$. The line marked $K_0 = 0.4$ represents the earth pressure at rest. All the test results showed that the lateral earth pressure was a maximum at some elevation above the dredge line. They also showed that the real pressure distribution depends on factors, such as the method of placing the fill, which do not receive any consideration in earth pressure theory. Hence, the agreement between the real pressure distribution and the Coulomb pressure distribution is by no means perfect because the intensity of the lateral earth pressure is a maximum at some elevation above the dredge line (Fig. 10 and Fig. 11) and not at the dredge line. The computation of the bending moments in the sheet piles on the basis of a Coulomb pressure with equal total intensity involves an error on the unsafe side.

Mr. Tschebotarioff also experimented with composite backfills, Fig. 11, Tests No. 4 and No. 5. The lateral pressure exerted by sand fills backed by clay fills in Test No. 4 was found to be slightly greater than the lateral pressure of a continuous sand backfill with identical properties. The lateral pressure exerted by sand fills located between the bulkhead and a clay slope as in Test No. 5 was slightly smaller than that exerted by the fill in Test No. 4. P. W. Rowe determined the distribution of the earth pressure on a flexible wall directly by means of pressure cells in 1952.[19] In agreement with Mr. Tschebotarioff's findings, he obtained the pressure distribution for dredge bulkheads as shown in Fig. 9(*c*). However, he found that an anchor yield of 0.1% of the height of the bulkhead is sufficient to change the pressure distribution into one that agrees fairly closely with the Coulomb theory. The test results are illustrated in Fig. 12(*a*). In Fig. 12, curve C_1 represents the distribution of the earth pressure on the inner face of the model bulkhead prior to yielding of the anchorage and C_0 represents distribution after the bulkhead had yielded. The total intensity of the pressure remained almost unchanged.

[19] "Anchored Sheet-Pile Walls," by P. W. Rowe, *Proceedings*, Inst. of C. E., Vol. I, January, 1952, pp. 27-70.

The anchorage of Mr. Rowe's model bulkhead was allowed to yield after the sand in contact with the upper part of the outer face of the bulkhead had been removed by excavation, whereas in practice the anchorage yields gradually during the process of excavation. This difference may involve a considerable difference in the type of pressure distribution. However, the yield of the anchorage may exceed considerably the limiting value of 0.001 H, and the pounding of waves or traffic vibrations may contribute further to a modification of the pressure distribution. Hence, even in cases of dredge bulkheads it does not seem justified to depend on the benefits to be derived from a difference between the real pressure distribution and the distribution computed on the basis of the Coulomb theory.

Curve C(Passive), Fig. 12(*a*), shows the results of the measurement of the passive earth pressure that acted upon the buried part of the model bulkhead. In order to obtain supplementary information concerning the effect of the type of wall movement on the distribution of the passive earth pressure, Mr.

Rowe experimented with a ⅜-in. steel plate that was buried to a depth of 2 ft in clean sand. The plate could be advanced toward the sand by rotation about a horizontal axis. The passive earth pressure on the wall was measured by use of seven pressure cells, spaced 3 in. on centers along the vertical axis of the area acted upon by the passive earth pressure. The test results are shown in Figs. 12(*b*), 12(*c*), and 12(*d*).

The pressure distribution represented by curve C(passive) in Fig. 12(*a*) is intermediate between those shown in Figs. 12(*c*) and 12(*d*), but none of them involves an increase of the passive earth pressure in simple proportion to the depth below the dredge line. With increasing flexibility of the buried part of the bulkhead, the movement of this part changes from a displacement almost parallel to the original position of the buried part into a movement by rotation about the lower edge of the bulkhead so that the distribution of the passive earth pressure becomes increasingly similar to that shown in Fig. 12(*b*), or Fig. 12(*c*).

Influence of Flexural Rigidity on Bending Moment

According to the theories of bulkhead analysis explained under the heading, "Classical Design Assumptions," and illustrated in Fig. 2, the conditions of end support and (as a consequence) the maximum bending moment in the sheet piles are independent of the flexural rigidity of the sheet piles. According to the same theories, the maximum bending moment decreases with increasing depth of sheet-pile penetration, whatever the flexural rigidity may be. These postulates are incompatible with what has been learned concerning the relation between horizontal displacement and horizontal soil reaction. The fallacies involved in the postulates have already been emphasized by Paul Baumann, M. ASCE, in connection with an analysis of the causes of the failure of Pier B in the Outer Harbor of the City of Long Beach, Calif.[20] In fact, if the sheet piles were perfectly rigid, the maximum bending moment would increase with increasing depth of pile penetration. However, no observational data were available concerning these important relationships until the results of Mr. Rowe's experimental investigations were published.[19]

[20] "Analysis of Sheet-Pile Bulkheads," by Paul Baumann, *Transactions*, ASCE, Vol. 100, 1935, pp. 707–797.

Mr. Rowe's model bulkheads consisted of metal plates of height H equal to from 20 in. to 36 in., and with different thicknesses. The dredge line (Fig. 13) was at a variable depth αH below the surface of the fill and the anchor line was at a depth βH. The vertical strains on the two surfaces of the metal plates were measured by strain gages spaced 2.5 in. vertically. The tests were performed using four different materials—coarse, clean sand; crushed rock (chips); pea gravel; and ashes. In one series of tests the materials were placed in a loose state and in a second series they were placed in a dense state. In each series of tests the extreme fiber stresses in the plate were measured by use of strain gages for different values of the free-height ratio α, anchor-level ratio β, surcharge ratio $\frac{q}{\gamma H}$, modulus of elasticity E of the wall, and the moment of inertia I of the cross section of the wall. The strain-gage readings furnished the data for computing the maximum bending moment in the plate

for each set of test conditions.

According to the results of Mr. Rowe's analytical studies, which preceded the tests, the condition for similitude between the bulkhead model and the prototype is satisfied if the values α, β, $\frac{q}{\gamma H}$ and ρ are the same. The symbol ρ indicates the flexibility number of the sheet piles,

$$\rho = \frac{H^4}{E I} \qquad \qquad (12)$$

in which I represents the rectangular moment of inertia of the piles. This conclusion is based on the tacit assumption that the modulus of elasticity of the sand increases in simple proportion to the depth below the dredge line. For loose sand, this assumption is at least approximately correct. For dense sand, the modulus of elasticity seems to increase more nearly with the square root of depth. Hence, if the sheet piles of a bulkhead are driven into dense sand, the conditions of end support will be less favorable than those of the model sheet piles embedded in sand with the same relative density.

The investigations led to the following conclusion, illustrated in Fig. 13. For very stiff bulkheads, the maximum bending moment M in the sheet piles is practically independent of the flexibility number and is equal to the value M(max) computed on the assumption of free earth support, Fig. 2(a). However, if ρ exceeds a certain value, the maximum bending moment M decreases with increasing values of ρ and finally approaches a value approximately equal to one-third of M(max). The critical flexibility ρ_c at which the maximum bending moment starts to drop below the value of M(max) increases with decreasing relative density of the sand, as shown in Fig. 13. The value ρ_c is almost independent of the values of α, β, and $\frac{q}{\gamma H}$ in the range over which these quantities are likely to vary under actual conditions.

The fact illustrated by Fig. 13—that the bending moment in sheet piles decreases with increasing flexibility of the piles—is chiefly the result of the interdependence between the type of deflection of the buried part of the sheet piles and the corresponding distribution of the passive earth pressure. If the sheet piles, with a depth of penetration D, were perfectly rigid and the anchorage unyielding, the buried part of the sheet piles would rotate about the anchor line. The corresponding distribution of the earth pressure would be similar to that shown in Fig. 12(d), and the center of the pressure would be located at an elevation of less than $D/3$ above the lower edge of the piles. This condition corresponds to the ideal "free earth support." As the flexibility increases, the outward movement of the lower edge of the piles becomes smaller and smaller. The yield assumes the character of a yield by rotation about the lower edge, involving a pressure distribution as shown in Figs. 12(b) and 12(c). The elevation of the center of the passive pressure increases to more than $D/2$, whereby the "free span"—equal to the distance between anchor line and center of the passive pressure—decreases, and the maximum bending moment decreases with the third power of the span. Finally, if the piles are extremely flexible, the lowest part of the sheet piles

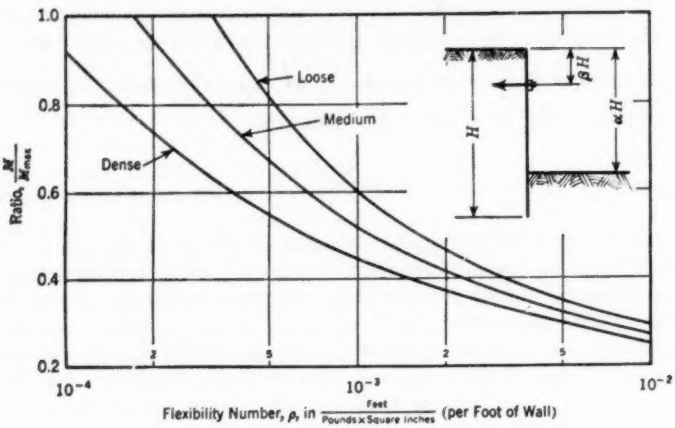


FIG. 13.—RELATION BETWEEN THE FLEXIBILITY NUMBER, ρ, OF SHEET PILES, AND BENDING-MOMENT RATIO, $\frac{M}{M(\max)}$ (LOGARITHMIC SCALE)

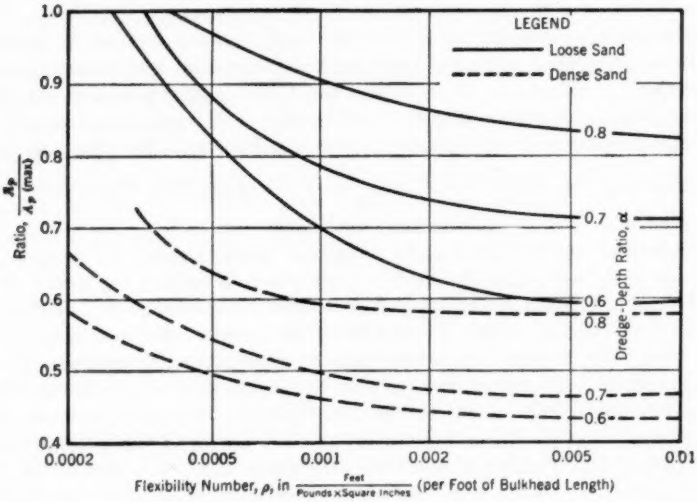


FIG. 14.—RELATION BETWEEN THE FLEXIBILITY NUMBER, ρ OF SHEET PILES, AND ANCHOR-PULL RATIO, $\frac{A_p}{A_p(\max)}$ FOR A β-VALUE OF 0.2

will neither advance nor rotate. In other words, the lower ends of the sheet piles will be "fixed" as shown in Fig. 2(*b*).

The critical value ρ_c of the flexibility number increases with increasing compressibility of the soil, because the resistance of the soil against tilt and outward movement of the buried part of the sheet piles decreases. This interdependence is illustrated in Figs. 12(*e*) to 12(*g*). If sheet piles are driven into peat (Fig. 12(*e*) they receive "free earth support" even if they are made of a flexible material such as wood.

If sheet piles are driven into silt or clay, the initial end restraint may be important enough to produce "fixed earth support." However, as time goes on, the soil yields under the lateral pressure because of progressive consolidation. The final lateral displacement of the buried part of the sheet piles can be even greater than that of sheet piles driven into loose sand. The yield is associated with a transition from fixed earth support to free earth support, whereby the maximum bending moment in the sheet piles increases. A permanent fixed earth support for piles driven into a cohesive soil can hardly be expected, unless the soil has been heavily precompressed by overburden pressures, that have subsequently been removed by natural processes such as erosion.

Approximate information concerning the influence of the flexibility of sheet piles on the maximum bending moment can be obtained by computation, using the theory of subgrade reaction. For the first time, this theory was applied to anchored bulkheads by Mr. Baumann in 1934.[20] Further contributions to this subject were made by H. Blum.[21] The sources of error involved in the procedure are rather important. They are common to all the computations based on the concept of subgrade reaction, such as the computations of the bending moments in nonuniformly loaded beams resting on an elastic subgrade.[22] Reliable methods for determining the coefficient of horizontal subgrade reactions are not available. In many instances the value of this coefficient has been seriously misjudged and the results of the computations were therefore misleading. The computations require considerable time and labor.

[21] "Beiträge zur Berechnung von Bohlwerken," by H. Blum, W. Ernst and Son, Berlin, Germany, 1951.

[22] "Theoretical Soil Mechanics," by Karl Terzaghi, John Wiley & Sons, Inc., New York, N. Y., 1943, pp. 345–346.

Anchor Pull

The decrease of the bending moments associated with an increase of the flexibility number (Fig. 13) results from a transition from the condition of free earth support (Fig. 12(*e*)), to the fixed end condition, Fig. 12(*g*). If the lowest part of the sheet piles is fixed, the fixed ends are acted upon by moments that carry part of the lateral pressure on the inner face, and as a consequence the anchor pull is reduced.

Mr. Rowe's test results concerning the influence of the flexibility number on the anchor pull are shown in Fig. 14 for a value of $\beta = 0.2$. In Fig. 14, the ordinates represent the measured anchor pull in percentage of the pull corresponding to free earth support. According to the test results, the relieving effect of the fixed end condition depends not only on the relative density of the backfill, as does the bending moment (Fig. 13), but also on the values of the anchor-level ratio β and the free-height ratio α. The investigation also

showed that the anchor pull decreases to some extent with increasing yield of the anchorage.

Since Mr. Rowe's data (illustrated in Fig. 13) have been published, there is no longer any justification for assuming fixed earth support without considering flexibility of the sheet piles.

The Shearing Resistance of Soils and the Angle of Repose

Whatever the design assumptions may be, the computation of the numerical values of the forces acting upon the bulkhead requires adequate knowledge of the shearing resistance of all the soils involved in the problem.

In the early days of bulkhead design, it was generally believed that the shearing resistance of a soil was equal to the normal pressure on the potential surface of sliding multiplied by the tangent of the slope angle at which the soil came to rest after it was dumped. This angle was called the angle of repose. For more than twenty years it has been known that there is no relationship between the shearing resistance and the tangent of the angle of repose except for clean, loose, and dust-dry sand. The angle of repose of any other material depends on both the characteristics of the material and the height of the slope. Therefore, it cannot be used as a basis for estimating the shearing resistance of the soil.

Whatever the characteristics of a soil may be, its shearing resistance can be divided into two components. One of them, commonly known as cohesion, is independent of the pressure, whereas the other one increases with increasing effective pressure $\bar{p}$ on the surface of sliding. The effective pressure $\bar{p}$ is equal to the difference between the total unit pressure p and the pore water pressure p_w. This fundamental relationship can be expressed approximately by the empirical equation:

$$s = c + (p - p_w) \tan \phi \ldots\ldots\ldots\ldots\ldots\ldots\ldots (13)$$

The limits of the validity of this equation are discussed in most references on soil mechanics.

For clean sand, $c = 0$ and $s = (p - p_w) \tan \phi$, in which s is the shearing resistance per unit of area. Cohesive soils, such as silt and clay, are commonly much more compressible than sand, and their permeability is very low. If the load on saturated, cohesive soil is increased, for example by the deposition of a backfill, the application of the load, p per unit of area, is associated with an equally important increase of the pore water pressure p_w, because the excess water drains out of the loaded soil stratum very slowly. The corresponding increase of the second term on the right-hand side of Eq. 13 becomes equal to zero and, as a consequence, the loaded stratum performs during and immediately after the application of the load as if its angle of internal friction were zero. Experience shows that the initial shearing strength of such soils is approximately equal to one half of their unconfined compressive strength q_u, and $s = \frac{q_u}{2}$.

If γ' is the submerged weight of a cohesive soil stratum in contact with a bulkhead and $\bar{p}$ is the effective unit load on the top surface of the stratum caused

by the submerged weight of the immersed part of the soil located above the stratum and the total weight of the materials located above the water table, the active earth pressure at any depth z below the surface of the cohesive stratum is

$$p_A = \bar{p} + \gamma' z - q_u \qquad (14)$$

per unit of area, and the passive earth pressure is

$$p_P = \bar{p} + \gamma' z + q_u \qquad (15)$$

Eqs. 14 and 15 were derived on the assumption that the surfaces of contact between soil and bulkhead are frictionless.[23] Because of the adhesion between the soil and the bulkhead, the values of p_A are somewhat smaller and those of p_P somewhat greater than those determined by Eqs. 14 and 15, respectively. Information concerning the values of q_u for clays with different consistencies is given in Table 1.

[23] "Soil Mechanics in Engineering Practice," by Karl Terzaghi and R. B. Peck, John Wiley & Sons, Inc., New York, N. Y., 1948, pp. 147–150.

TABLE 1.—UNCONFINED COMPRESSIVE STRENGTH OF SILT AND CLAY

Consistency (the drillman's designation)	Compressive Strength, q_u, in Tons per Square Foot	
	Minimum	Maximum
Very soft	0	0.25
Soft	0.25	0.50
Medium	0.50	1.00
Stiff	1.00	2.00
Very stiff	2.00	4.00
Hard	4.00	∞

In the course of time, the shearing resistance of cohesive soils such as silt or clay increases because of progressive consolidation. Nevertheless, this process is harmful because the consolidation is associated with an increase of the maximum bending moment in the sheet piles, as previously explained under the heading, "Influence of Flexural Rigidity on Bending Moment."

If a cohesive soil, such as fine silt or clay, is deposited under water, consolidation hardly starts during construction. At any depth below the surface, the pore water pressure p_w almost equals the overburden pressure p at that depth, so that $(p - p_w)$ in Eq. 15 equals zero. Since the cohesion c of such backfills is also very small, the lateral earth pressure exerted by such a backfill and its resistance against lateral displacement are at any point equal to the vertical pressure in the fill at that point; hence,

$$K_A = K_P = 1.00 \qquad (16)$$

Because of the absence of any definite relationship between the angle of repose and the shearing resistance of soils, reliable information concerning the active and passive earth pressure of the different soils in contact with an anchored bulkhead can be obtained only on the basis of: (1) The results of laboratory tests simulating the conditions under which the soils will be subject

to shear in the field, or (2) empirical values derived from the results of tests such as those which will be proposed in Part II of this paper.

II. DESIGN OF ANCHORED BULKHEADS

Uncertainties Involved in the Design of Anchored Bulkheads

The experimental investigations described in Part I have made it possible to eliminate the most serious misconceptions associated with the customary methods of bulkhead design. On the basis of the findings, one can reliably estimate the forces exerted on anchored bulkheads by homogeneous layers of soil with known physical properties. Hence, the uncertainties involved in the design of bulkheads no longer result from inadequate knowledge of the fundamental principles involved. They are caused only by the fact that the structure of natural soil deposits is usually complex, whereas the theories of bulkhead design inevitably presuppose homogeneous materials. Not even the backfills composed of excavated and transported soils can be considered homogeneous. Because of local variations of the soil properties within the borrowpit area and segregation according to grain size during the process of underwater deposition, the characteristics of the backfill may change from place to place and its properties cannot be reliably determined by tests in advance of construction.

Because of these conditions, the most economical and expedient procedure consists in estimating the constants and coefficients—such as the unit weights and the coefficients of earth pressure—on the basis of the results of exploratory borings and of routine tests performed on representative samples, and compensating for the uncertainties involved in this procedure by an adequate margin of safety. More elaborate investigations are justified only in exceptional cases.

General Design Procedure

The design of anchored bulkheads requires several successive operations: (*a*) Evaluation of the forces that act on the inner face of the bulkhead, (*b*) determination of the depth of sheet-pile penetration, (*c*) computation of the maximum bending moments in the sheet piles, (*d*) evaluation of the anchor pull, and (*e*) selection of allowable stresses in the construction materials in accordance with the uncertainties involved in the evaluation of the acting forces.

Each operation is described herein under a separate subheading. The essential data required for performing the operations are obtained by exploratory borings at the site of the bulkhead and in the borrowpit area and by certain routine tests as specified subsequently.

At the site of the bulkhead, the borings must be supplemented by the standard penetration test[24] or an equivalent procedure to obtain information

[24] "Soil Mechanics in Engineering Practice," by Karl Terzaghi and R. B. Peck, John Wiley & Sons, Inc., 1948, p. 265.

concerning the relative density of the strata of sand or of silty sand that will be in contact with the sheet piles. Drive-samples of the sand are secured and a mechanical analysis is performed. The results define the general nature of

the cohesionless strata. If layers of silt or clay are encountered, fairly undisturbed samples should be recovered and the natural water content, Atterberg limits, and the unconfined compressive strength q_u of these samples should be determined. If possible, the boring operations should be supplemented by field tests with the vane borer. This method of testing was developed in Sweden and it has been extensively used in the United States and in Canada.[25]

[25] "The Vane Borer," by L. Cadling and S. Odenstad, *Proceedings*, Royal Swedish Geotechnical Inst., No. 2, 1950; Stockholm, Sweden.

The value of the natural water content is required for computing the full and the submerged unit weight, γ and γ', respectively, of the materials and the value of the unconfined compressive stress q_u determines, according to Eqs. 14 and 15, the active and passive earth pressure of clay and silt.

The soils encountered at the bulkhead site and the backfill materials are classified on the basis of the following system: Clean sand (dense, medium,or loose); silty sand (dense, medium, or loose); and silt or clay (hard to very soft, as in Table 1). A sand should be classified as silty, if more than 5% passes the 200-mesh sieve.

Forces Acting on Inner Face of Bulkheal

Varieties of Forces.—Fig. 15 shows diagrammatic sections through bulkheads ab. The sheet piles are driven into sand (Fig. 15(*a*)) and clay (Fig.15(*b*)), respectively. Both bulkheads are backfilled with clean sand which has been sluiced into place.

The intensity of the forces acting on the inner face of each bulkhead is represented by the width of the pressure areas, numbered from I to IV on the left-hand side of ab. These forces include the following:

I. Active earth pressure produced by the weight of the backfill;
II. Active earth pressure produced by the uniformly distributed surcharge, q;
III. Unbalanced water pressure (see also Fig. 5); and
IV. Lateral pressure caused by line load q'.

The computation of the intensity of these forces must be preceded by an evaluation of the full and submerged unit weights of the various soils in contact with the inner face and of their coefficients of active earth pressure. In each figure the symbol $\bar{p}$ with subscfipts indicates the effective vertical unit pressure on the different horizontal sections through the soil.

The active earth pressure exerted by the backfill depends on the coefficient of active earth pressure K_A of the fill material and on its effective unit weight. It may be increased temporarily or permanently by a uniformly distributed surcharge, by line loads, or by mobile or stationary concentrated loads.

Unit Weights.—The effective unit weight depends on the position of the soil with reference to the water table. Above the water table, it is equal to the sum of the dry weight of the soil and the weight of the water contained in the voids (moist unit weight). Below the water table it is equal to the submerged weight of the soil particles. Limiting values for the moist and the submerged unit weights for cohesionless soils are given in Table 2. In the compu-

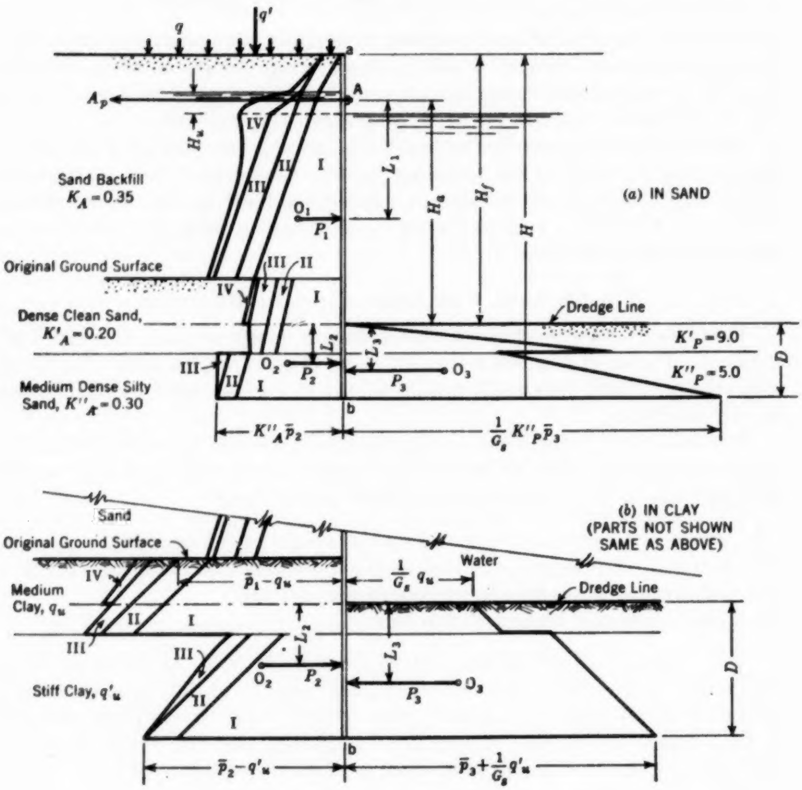


FIG. 15.—FORCES ACTING ON BULKHEADS CONSISTING OF SHEET PILES DRIVEN INTO SAND AND INTO CLAY

TABLE 2.—UNIT WEIGHTS OF SOILS, AND COEFFICIENTS OF EARTH PRESSURE

Type of soil	Unit Weight[a] of Moist Soil, γ		Unit Weight[a] of Submerged Soil, γ'		Coefficient of Active Earth Pressure, K_A				Coefficient of Passive Earth Pressure, K_P		
	Minimum	Maximum	Minimum	Maximum	For backfill	For soils in place	Friction Angles[b] ϕ	Friction Angles[b] δ	For soils in place	Friction Angles[b] ϕ	Friction Angles[b] δ
(1)	(2)	(3)	(4)	(5)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
Clean Sand:											
Dense	110	140	65	78		0.20	38	20	9.0	38	25
Medium	110	130	60	68		0.25	34	17	7.0	34	23
Loose	90	125	56	63	0.35	0.30	30	15	5.0	30	20
Silty Sand:											
Dense	110	150	70	88		0.25			7.0		
Medium	95	130	60	68		0.30			5.0		
Loose	80	125	50	63	0.50	0.35			3.0		
Silt and clay[c]	$\frac{165\,(1+w)}{1+2.65\,w}$		$\frac{103}{1+2.65\,w}$		1.00	$1-\frac{q_u}{\bar{p}+\gamma z}$			$1+\frac{q_u}{\bar{p}+\gamma z}$		

[a] In pounds per cubic foot. [b] These angles, expressed in degrees, are ϕ, the angle of internal friction, and δ, the angle of wall friction, and are used in estimating the coefficients under which they are listed. [c] The symbol γ represents γ or γ', whichever is applicable; $\bar{p}$ is the effective unit pressure on the top surface of the stratum; q_u is the unconfined compressive stress; w is the natural water content, in percentage of dry weight; and z is the depth below the top surface of the stratum.

tation of the active earth pressure, the upper limiting values should be used unless the real unit weights have been estimated on the basis of test results. The unit weight of cohesive soils is determined by their natural water content w and it can be computed by use of the equations given in Table 2.

Coefficients of Active Earth Pressure.—Design values for the coefficient of active earth pressure are in Table 2. The backfills are usually deposited underwater and their structure is likely to be loose. The deflection of the bulkhead is not sufficient to mobilize the full value of the shearing resistance of the backfill (as shown in Fig. 3). Therefore, the values of K_A assigned to the backfill materials are higher than the corresponding values for the same materials in place.

The values of K_A for cohesionless soils in place were estimated on the basis of the ϕ-values and δ-values in Table 2. In selecting values of δ, the angle of wall friction, it was reasoned that, since the active earth pressure is equal to the sum of the anchor pull and passive earth pressure (as shown in Fig. 2(*a*)), the total value of the active earth pressure is considerably greater than that of the passive earth pressure. Hence, for any specified value of the angle of wall friction, the resultant of the friction forces would tend to pull the sheet piles down. If the lower edge of the sheet piles were rigidly supported, the resultant would be carried by this rigid support. In reality, the resistance against a downward movement of the lower edge of the sheet piles is small. Therefore, the sheet piles will settle so that the total wall friction on the inner face will decrease until it becomes nearly equal to the friction force on the outer face. Because of this condition, the angle of wall friction for the active pressure

was assigned smaller values than that for the passive earth pressure.

The K_A-value for dense sand can be much lower than the value of 0.2 given in Table 2, but this is only a possibility, because the value of K_A depends, to a large extent, on the uniformity of the sand and the shape of the grains.[3] Therefore, the use of a lower value would be justified only if it has been computed on the basis of test results on representative samples.

Silty sand has been assigned greater K_A-values than clean sand with equal relative density because its angle of internal friction is likely to be smaller and its compressibility higher than that of clean sand with equal relative density. Furthermore, the evaluation of the relative density of silty sands by use of the standard penetration test appears to be much less reliable than that of clean sands. The K_A-values for silt and clay are represented in Table 2 by an equation that gives results which are on the safe side becuase the pressure reducing effect of the adhesion between soil and sheet piles has been disregarded.

Uniformly Distributed Surcharge.—The lateral unit pressure resulting from a uniformly distributed surcharge, q per unit of area, is at any depth equal to q times the value of K_A for that depth (area II in Fig. 15).

Unbalanced Water Pressure.—The first step in studying the unbalanced water pressure is to estimate the greatest hydraulic head, H_u in Fig. 5(a), which can be anticipated at the site of the bulkhead. This can be done on the basis of regional hydrographic data such as local tidal curves or flood records. If the coefficient of permeability of all the soils in contact with the bulkhead is of the same order of magnitude, the unbalanced water pressure associated with a head H_u can be estimated on the basis of the assumption represented by the broken line cde in Fig. 5(b). Area III in Fig. 15(a) has been plotted on this assumption. Otherwise, the effects of stratification on the distribution of the unbalanced water pressure must be considered. In any event, precautions must be taken to prevent the unbalanced water pressure from exceeding the estimated value even under exceptional conditions such as an outgoing spring tide combined with a heavy rainstorm. This may, for example, be accomplished by adequate surface drainage.

If the bulkhead is backfilled with a material—such as silt or soft clay—which does not begin to consolidate during construction, the initial water table in the backfill is located at the surface of the backfill. The coefficient of active earth pressure of such materials is equal to unity (Table 2). Therefore, the total lateral pressure of the backfill, earth and water pressure combined, is equal to the fluid pressure of a liquid the unit weight of which is equal to that of the saturated backfill material.

Line and Point Loads.—Line loads and point loads can be established only on the surface of backfills with adequate bearing capacity. Otherwise the loads must be carried by piles, and pile-supported loads have no influence on the lateral pressure.

Line loads are stationary and the lateral pressure resulting from the loads is determined by Eqs. 4 and 5. Point loads are either stationary or mobile. The wheel loads of cranes that can be displaced on rails along the waterfront constitute mobile point loads. The unit pressures produced by a point load along the line of intersection ab (Fig. 7(a)) between the bulkhead and a vertical plane through the load at right angles to the bulkhead are determined by

Eqs. 9 and 10. A mobile point load may occupy any position on the line along which it travels. Hence, every sheet pile may be acted upon by these pressures. However, if the point load is stationary the unit pressures produced by the load decrease on both sides of the line ab, as shown in Fig. 7(*a*). Upper limiting values for these pressures are obtained by use of Eq. 11 (Fig. 8(*b*)).

Figs. 6(*a*) and 8(*a*) show that the computed unit pressures on the lower part of the lateral supports are consistently greater than the real pressures. As a partial compensation for this difference, it should be assumed that the lateral pressure produced by line loads and point loads decreases, at the dredge line, to zero. Hence, in Eqs. 4, 5, 9, and 10 the value H should be made equal to the vertical distance H_f between the dredge line and the surface of the backfill. In Fig. 15 the lateral pressure caused by point loads or line loads is represented by area IV.

Depth of Sheet-Pile Penetration

Factors Determining Penetration Depth.—The investigations of Mr. Rowe have shown that no tangible benefits can be obtained by driving the sheet piles deeper than the depth required to assure an adequate margin of safety with respect to a failure resulting from an outward movement of the buried part of the sheet piles, and a sufficiently small horizontal displacement of the lower edge of the sheet piles.[19]

The resistance of a cohesionless soil against an outward movement of the buried part of the sheet pile depends on its effective unit weight and on the coefficient of passive earth pressure. The resistance of a cohesive soil such as silt or clay depends only on its unconfined compressive strength.

Unit Weights.—Because the passive earth pressure of a cohesionless soil decreases with decreasing unit weight, the computations should be made on the basis of the lower limiting values in Table 2, unless more accurate values have been obtained by tests on representative samples. Should the water table in the backfill be temporarily located above the free water level, as in Fig. 5(*a*), the reduction of the effective unit weight resulting from the seepage pressure associated with upward percolation through the soil in contact with the outer face of the bulkhead must be considered. If the coefficient of permeability of all the soils in contact with the bulkhead is of the same order of magnitude, the effective unit weight of the soil acted upon by the seepage pressure can be estimated by use of Eq. 3. Otherwise, an appropriate value for the hydraulic gradient i in equation Eq. 2 must be selected on the basis of the soil profile. In many instances the gradient i is too small to require any consideration.

Coefficients of Passive Earth Pressure.—Lower limiting values for the coefficients of passive earth pressure are in Col. 11, Table 2. The values of ϕ and δ used in the estimates of K_P-values for clean sand (Fig. 4) are in Cols. 12 and 13. The K_P-value for a well-graded dense sand or a mixture of sand and gravel can be almost twice as high as the tabulated value for dense sand. This, however, is only a possibility. Hence, higher values should be tolerated only if they are based on test results.

In the rare event that the buried part of an anchored bulkhead derives its lateral support from a sand fill, the sand can be assigned a K_p-value of 3.

Silty sand has been assigned smaller K_P-values than clean sand with equal relative density. This was done because of the conditions previously explained (under the heading, "Forces Acting on Inner Face of Bulkhead: Coefficients of Active Earth Pressure"). Very loose, silty sand cannot be expected to provide adequate lateral support for the buried part of sheet piles because of its high compressibility.

If the surface of the soil constituting the lateral support of the buried part of the bulkhead is not horizontal, the passive earth pressure must be determined by a graphical procedure on the basis of the ϕ-values and δ-values in Table 2. Since the assumption of a plane surface of sliding may involve important errors on the unsafe side (Fig. 4(*b*)), the logarithmic spiral method should be used.[25]

[25] "Theoretical Soil Mechanics," by Karl Terzaghi, John Wiley & Sons, Inc., New York, N. Y., 1943, pp. 100–113.

Computation of Depth of Sheet-Pile Penetration

The general principles in the computation of penetration depth are illustrated in Fig. 15. The area on the left-hand side of the bulkhead ab represents the active pressures on the inner face, and the area on the right-hand side represents the passive earth pressures, plotted on the assumption of free earth support.

The actual distribution of the passive earth pressure under conditions of free earth support (Fig. 12(*a*)) is approximately trapezoidal. However, the assumption of a trapezoidal distribution would complicate the computations considerably except in the rare event of a bulkhead with sheet piles driven into homogeneous ground. Therefore, the Coulomb concept is retained. In accordance with this concept, the passive earth pressure of homogeneous material increases in a manner similar to hydrostatic pressure (in simple proportion to the depth below the surface). The error resulting from this simplifying assumption is unimportant and on the safe side.

In order to provide for an adequate margin of safety with respect to a failure of the lateral earth support of the buried part of the sheet piles, the pressure areas on the right-hand side of line ab in Fig. 15(*a*) are constructed on the assumption that the coefficient of passive earth pressure is equal to the K_P-values in Table 2, divided by a safety factor G_s. If the sheet piles are driven into silt or clay, as in Fig. 15(*b*), the safety requirements are satisfied by assigning to the soil in contact with the outer face of the bulkhead a reduced unconfined compressive strength, $\frac{1}{G_s} q_u$. The conditions requiring consideration in the choice of G_s will be discussed, subsequently, under the heading, "Safety Requirements."

In Fig. 15, points O_1, O_2, and O_3 represent the centers of gravity of the pressure areas above the dredge line (O_1), and below the dredge line on the active side (O_2) and passive side (O_3) of the bulkhead. The total pressures represented by these areas are P_1, P_2, and P_3. The position of their lines of action are defined by the distances L_1, L_2, and L_3. The values of L_1 and P_1 are

independent of the depth D of sheet-pile penetration. The depth D must satisfy the condition that the sum of all the moments about the anchor point A is equal to zero,

$$P_1 L_1 + P_2(H_a + L_2) = P_3(H_a + L_3) \quad \ldots\ldots\ldots\ldots (17)$$

If the values of P_2, P_3, L_2, and L_3 are expressed in terms of the unknown quantity D, an equation of the third degree is obtained which can be solved for D.

Computation of Maximum Bending Moments

Free Earth Support.—If the profile of the soil into which the sheet piles will be driven is erratic or if no reliable data concerning the details of the soil profiile are available, the maximum bending moment in the sheet piles should be computed on the assumption of free earth support. The acting forces are shown in Fig. 15(*a*) for bulkheads with lateral sand support and in Fig. 15(*b*) for bulkheads with clay support. The maximum bending moment can be determined by analytical or graphical methods in the usual manner.

Mr. Tschebotarioff has suggested computation of the maximum bending moment in the sheet piles on the assumption of fixed earth support irrespective of flexibility and of the relative density of the sand into which the sheet piles are driven.[18] Mr. Rowe showed that the errors involved in such a procedure can be very important and they are on the unsafe side.[19] Moment reduction caused by partly or completely fixed earth support can only be expected under the conditions to be described subsequently.

Moment Reduction Resulting from Flexibility Effects.—If the sheet piles are to be driven into a fairly homogeneous stratum of clean sand with known relative density, the maximum bending moment for free earth support can be reduced on the basis of the results of Mr. Rowe's investigations that are represented in Fig. 13. As an example, the reduction of the bending moments in sheet piles driven into sand with medium density will be considered.

The first step consists of computing the maximum bending moments for free earth support, M(max.), and of the cross section of the piles required to withstand M(max). The conditions that must be considered in choosing the allowable fiber stresses f will be discussed subsequently (under the heading, "Safety Requirements"). The flexibility number ρ_1 of these piles is determined by Eq. 12.

At a specified value of M(max) the moment of inertia I and the corresponding flexibility number ρ_1 depend on the value assigned to the allowable fiber stresses and on the construction material. Hence, at a specified value of M(max), very different values of ρ_1 may be obtained, such as ρ_{1t} for timber piles, ρ_{1s} for steel piles and ρ_{1c} for reinforced concrete piles.

The next step consists in plotting the moment-reduction curve, which is shown as the heavy curve in Fig. 16. This curve was obtained by multiplying the ordinates of the curve for sand with medium density in the Rowe diagram (Fig. 13) by M(max). The abscissa ρ_c of point C at which the curve starts to descend represents the critical flexibility number. If the flexibility number of

the pile required by M(max) is smaller than ρ_c, the maximum bending moment in this pile is determined by the condition of free earth support and is equal to M(max).

In Fig. 16 the sheet piles with flexibility numbers ρ_1, such as ρ_{1t}, ρ_{1s}, and ρ_{1c}, are represented by points S. At a given value of M(max), the flexibility number of timber sheet piles with the required moment of inertia has the greatest value—designated as point S(timber)—and that of reinforced concrete piles the smallest value—designated as point S(concrete). Point S(steel), representing steel sheet piles, occupies an intermediate position.

If point S is located on the left-hand side of C, no moment reduction can be tolerated. A position of point S on the right-hand side of C indicates that the pile represented by the point is stronger than necessary because the maximum bending moment in the pile M_1 will be less than M(max). In order to select a more economical profile for the sheet piles, the allowable bending moments M', M'', $\cdots$ and the corresponding flexibility numbers ρ', ρ'', $\cdots$ for various weaker profiles are computed. In Fig. 16 these weaker sheet piles are represented by points such as S′(steel) with the ordinate M' and the abscissa log ρ'. All these points are located in proximity of a curve that intersects the moment-reduction curve at N, with abscissa log ρ_a and ordinate M_a. The corresponding moment reduction is M(max) $- M_a$. However, because of the rudimentary state of present (1953) knowledge of the performance of bulkheads under field conditions, the computed bending moment M(max) should be reduced by not more than $\frac{1}{2}(M(\text{max}) - M_a)$.

If the sheet piles are to be driven into a homogeneous stratum of dense or medium silty sand, Mr. Rowe's moment-reduction curves for medium and loose sand should be used instead of those for dense and medium sand. Sheet piles to be driven into loose, silty sand should be dimensioned for free earth support because the compressibility of such sands may be very high.

Sheet Piles Driven into Silt or Clay.—In Part I it was shown that the initial earth support for sheet piles driven into silt or clay is likely to be fixed. However, as time elapses, the end restraint decreases because of progressive consolidation of the soil resisting the lateral pressure of the buried part of the bulkhead. As the present (1953) state of knowledge concerning the effect of the progressive yield on the end restraint is limited, no moment reduction should be tolerated.

Anchor Pull

Anchor Pull for Free Earth Support.—The anchor pull A_p for free earth support is determined by the condition that the sum of all the horizontal forces that act on the bulkhead (Fig. 15), is equal to zero:

$$A_p = (P_1 + P_2 - P_3)\, l \qquad (18)$$

in which l is the spacing between anchor rods.

The force P_1 also includes the lateral pressure caused by mobile or stationary point loads. In Fig. 15 this pressure is represented by area IV, that is equal to the lateral pressure per unit of length of the bulkhead along section ab in Fig. 7(a). According to this figure, the lateral pressure produced by the

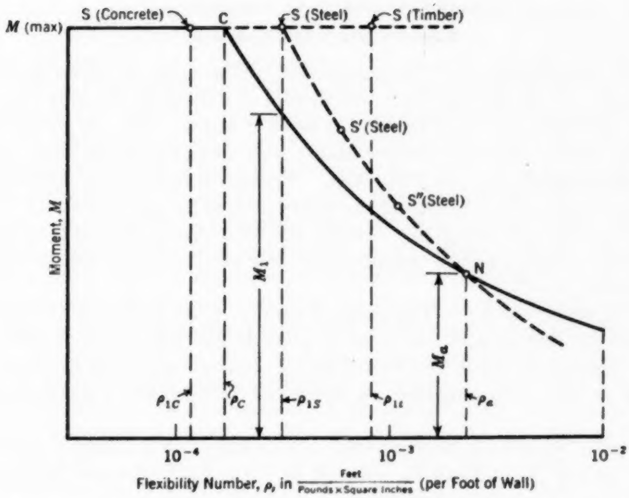


FIG. 16.—GRAPHIC PROCEDURE FOR DETERMINING TOLERABLE MOMENT REDUCTION

point load decreases from ab in both horizontal directions. Hence, if the backfill carries a point load, computation of A_p by use of Eq. 18 involves an error on the safe side. The importance of the error can be estimated and corrected on the basis of the experimental data shown in Figs. 7 and 8. Usually the error is too small to require consideration.

Anchor-Pull Reduction.—As indicated in Fig. 14, the anchor pull A_p decreases with increasing flexibility number, but the decrease is not as important as the corresponding decrease of the maximum bending moment M_1 in Fig. 13. The anchor pull also depends on several factors other than the properties of the backfill material and the flexibility number. Therefore the anchor pull should be computed on the assumption of free earth support.

Safety Requirements

General Considerations.—The importance of the error involved in estimating the forces which act on the inner face of a bulkhead depends to a large extent on the degree of uniformity of the fill material, the complexity of the soil profile, and the amount of information which can be, or has been, secured concerning the significant soil properties. Therefore, it would be uneconomical to establish a rigid code concerning the safety factors and the allowable stresses. Designers who are not thoroughly familiar with the principles and techniques of soil mechanics are advised to consider only free earth support and to use large safety factors.

Passive Earth Pressure.—The safety factor G_s (Fig. 15(*a*)), with respect to a failure of the lateral earth support of piles driven into clean or silty sand, should be assigned a value between 2 and 3, depending on the accuracy with which the active earth pressure on the inner face of the bulkhead can be estimated.

If the earth support is derived from silt or clay, the limiting values for G_s (as shown in Fig. 15(*b*)) can be reduced to 1.5 and 2.0 because the K_P-value defined by Eq. 15 is on the safe side.

Whatever the subsoil conditions may be, the computed depth of sheet-pile penetration should always be increased by 20% as insurance against the effects of unintentional excess dredging, unanticipated local scour, and of the presence of pockets of exceptionally weak material in the zone of passive earth pressure which have not been revealed in the borings. The corresponding increase of the cost of the bulkhead is small as compared to the increase of the margin of safety for all parts of the bulkhead associated with the excess penetration. The bending moments in the sheet piles and the anchor pull should be determined on the basis of the computed depth—and not of the increased depth—of sheet-pile penetration.

Allowable Fiber Stresses in Sheet Piles.—The maximum bending stresses in the sheet piles of anchored steel bulkheads backfilled with clean sand can be made equal to at least two-thirds of the yield point stress f_y. This recommendation also applies to dredge bulkheads supporting clean sand in place. If the backfill consists of hydraulically excavated, silty sand or a mixture of clean and silty sand, the real value of the coefficient of earth pressure can locally be considerably higher than the values given in Table 2 for silty sand because of the inevitable segregation of fine and coarse material during the process of deposition. Therefore, the fiber stress in the sheet piles of steel bulkheads backfilled with such materials should be assigned a value of not

more than two thirds of the yield point. However, if a bulkhead is to be backfilled with hydraulically excavated clay or silt to which a coefficient of active earth pressure equal to unity is assigned, the sheet piles can safely be dimensioned on the basis of $f = f_y$, becuase the real lateral earth pressure cannot be greater than that which is computed.

The extreme fiber stresses in sheet piles having locks on the neutral axis depend largely on lock friction. Little information is available (1953) concerning the values that can safely be assigned to the lock friction in straight rows of sheet piles. Hence, it is advisable that such sheet piles be dimensioned on the assumption that the locks are lubricated. The flexibility number should be computed on the same basis. At a given value of M(max), the flexibility number for sheet piles dimensioned on the assumption of lubricated locks is considerably greater than the corresponding number for piles with "frozen locks." Therefore, a decrease of the friction in the interlocks can be associated with a decrease of the maximum bending moment in the sheet piles.

The allowable stresses for construction materials other than steel should be selected on the basis of the considerations that have been set forth for steel.

Allowable Stress in Anchor Rods.—Eq. 18, which is used for computing the anchor pull, involves the assumption that the distribution of the active earth pressure on the inner face is in accordance with the Coulomb theory. The real distribution may be somewhat different—as shown in Figs. 11 and 12—and the corresponding anchor pull may be greater than the computed anchor pull. Furthermore, if the uppermost part of the sheet piles is in contact with a soil having low compressibility and the lower part moves out (for example because of progressive consolidation of the soil that provides the lateral earth support) the anchor pull increases. The anchor pull may also increase because of repeated application and removal of heavy surcharges. Finally, an unequal yield of adjacent anchorages produces an increase in the anchor pull in some anchor rods, associated with a decrease in others. Because of these possibilities, the anchor rods should be dimensioned on the basis of more conservative allowable stresses than are applied to the design of sheet piles.

Supplementary Safety Provisions.—The first and foremost requirement for the success of a bulkhead project is avoiding, during construction, loading conditions which the designer has not anticipated. This vital aspect of the bulkhead problem has been discussed by J. R. Ayer, M.ASCE, and R. C. Stokes.[27]

[27] "The Design of Flexible Bulkheads," by J. R. Ayer and R. C. Stokes, *Proceedings-Separate No. 166*, ASCE, January, 1953.

In addition to providing for an adequate margin of safety in the design and for protection of the construction materials against deterioration, it is also necessary to protect the bulkhead against the consequences of processes and events beyond the scope of theory.

If a bulkhead is at the seashore or at the bank of a swiftly flowing river, consideration should be given to the possibility that the scour along the foot of the exposed part of the outer face of the sheet piles may increase considerably the free height of the bulkhead, H_f (Fig. 15). The upper limiting value for the increase of H_f should be estimated on the basis of experience with local scouring action.

After the sheet piles for a fill-bulkhead with submerged outer face are

driven, a careful inspection should be made of the condition of the locks or joints. If gaps between adjacent sheet-piles are found, they should be plugged by divers. A case of important loss of soil caused by a flow of sand through gaps between sheet piles was reported by J. C. Gebhard.[28]

[28] "Cave-Ins of Sandy Backfills," by J. C. Gebhard, *Transactions*, ASCE, Vol. 114, 1949, pp. 490–498.

If the backfill material is very compressible or if the subsoil of the backfill contains layers of soft silt or clay, it is necessary to encase the anchor rods in large conduits with a circular or rectangular cross section that can follow the downward movement of the backfill without transmitting vertical forces onto the anchor rods. As an alternative, the rods could be supported at several points by piles, but then allowance must be made for the stresses in the tie rods caused by the earth load which acts on the rods between the points of support. On Pier C at Long Beach, it was found that the anchor rods embedded in the backfill carried a large part of the weight of the backfill lying above the level of the anchor line. This load increased considerably the tension in the rods.[29]

[29] "Field Study of a Sheet-Pile Bulkhead," by C. Martin Duke, *Proceedings-Separate No. 155*, ASCE, October, 1952.

Bulkhead Failures.—All the bulkhead failures that have come to the writer's attention can be attributed to one of two causes. The designer has estimated earth pressure and earth resistance on the basis of the "angle-of-repose" concept, or else he has failed to notice a source of weakness in the ground below the level of the lower edge of the sheet piles.

It is known that there is no relationship between the angle of repose of a soil and its angle of internal friction except in the case of loose and dust-dry sand. Nevertheless, several bulkhead computations have been made in which soft clay was assigned an angle of internal friction of 11°, on the strength of the observation that the front part of a sheet of hydraulic-fill clay commonly has a slope approximating one on five. The bulkheads to which these computations referred have failed.

Some bulkheads with sheet piles driven into sand have failed because of an outward movement of both bulkhead and fill on a soft clay stratum beneath the sand. The design of the bulkheads and of the anchorage was satisfactory, but the designers failed to take heed of the subsoil of the stratum that provided the lateral support for the buried part of the sheet piles.

If the submerged natural earth in front of a bulkhead slopes downward in an offshore direction, the slope may fail along a surface of sliding located below the anchor and the lower edge of the sheet piles. Several bulkhead failures of this kind have occurred on the Whangpoo River at Shanghai, China. The subsoil consisted of river silt, the offshore slope was about one on two and most of the failures occurred within two hours after low tide following spring tide.

At the present state of knowledge of the physical properties of soils (1953), all the aforementioned bulkhead failures could have been avoided by proper consideration of the results of exploratory borings and of a few identification tests on representative samples of the earth materials involved.

Conclusions

1. Since the time (1910 to 1930) when the current procedures (1953) for bulkhead computations originated, knowledge of the physical properties of earth materials and of the mechanics of earth pressure has increased vastly, yet the procedures remained practically unaltered. This paper contains suggestions for revisions on the basis of the experimental and observational data which have been secured during the last few decades.

2. Many bulkheads have been designed without adequate information concerning the relative density of the sand layers and the shearing strength of clay strata in contact with the bulkheads. Bulkheads designed in such a manner can be badly overdimensioned or they may be unsafe, depending on factors unknown to the designers. One of the most common causes of faulty design is the erroneous assumption that the shearing resistance of cohesive soils can be evaluated on the basis of an angle of repose.

3. The importance of the errors involved in the estimate of the soil constants appearing in the equations depend to a large extent on the complexity of the structure of the strata into which the sheet piles are driven, on the degree of uniformity of the material in the borrowpit area, and on the quality of the subsoil exploration. Therefore, it would be unwarranted to establish rigid rules for the factors of safety that should be used. One of the responsibilities of the designer is to evaluate the prevailing uncertainties and to choose the factors in accordance with his findings. Limiting values are proposed in this paper.

4. No bulkhead theory can possibly anticipate all the varieties of subsoil and hydraulic conditions that may be encountered in practice, and every case requires a certain amount of independent judgment. Hence, if the subsoil conditions do not conform to a standard pattern, designers who are not thoroughly familiar with the basic principles and techniques of soil mecahnics are advised to assume free earth support and to use conservative factors of safety. When solving unusual problems, the designer should consult the observational data (summarized in Part I) on which the design procedures are based.

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The author expresses his thanks to A. E. Cummings, M. ASCE, S.D. Wilson, A.M. ASCE, and S. B. Avery, Jr., for valuable comments and sugsestions. He is also indebted to Mr. Rowe who kindly placed at his disposal copies of the original records of various tests.

APPENDIX. LIST OF SYMBOLS

The following letter symbols conform essentially with ASCE *Manual of Engineering Practice No. 21* ("Soil Mechanics Nomenclature") and American Standard Letter Symbols for Structural Analysis (ASA Z10.8—1949) prepared by a Committee of the American Standards Association, with ASCE participation, and approved by the Association in 1949:

A_p = anchor pull;
D = depth of sheet-pile penetration;
d = the ration of deflection to the height of a wall;

E = the modulus of elasticity;
f = the allowable stress in bending of the piles;
f_y = the yield point;
G_s = the factor of safety;
H = the height of a lateral support, total length of sheet piles;
H_a = the vertical distance from the anchor to the dredge line;
H_f = the vertical distance between the dredge line and the surface of the backfill;
H_w = the vertical distance between the free water level and the water table in the backfill;
I = the moment of inertia of the cross section of a sheet pile;
i = the hydraulic gradient;
K_A = the coefficient of active earth pressure (the ratio between the normal component of the earth pressure on the lateral support and the corresponding fluid pressure);
K_P = the coefficient of passive earth pressure;
K_0 = the pressure coefficient for earth at rest;
L_1, L_2, and L_3 = the vertical distances from centers of pressure;
l = the anchor spacing;
M = the maximum bending moment in a sheet pile:

- M(max) = the maximum bending moment in a sheet pile computed on the assumption of free earth support;
- M' = the allowable bending moment for a sheet pile with a given flexibility number;
- M_1 = the maximum bending moment in a sheet pile with a given flexibility number;
- M_a = the maximum bending moment in a sheet pile with flexibility number ρ_a.

m = the ratio between the horizontal distance from the wall to the height of the wall;
n = the ratio between the depth below the surface of the backfill and the height of the wall;
P = the total normal pressure on a lateral support produced by a point load;
P_w = the unbalanced water pressure;
p = the unit pressure:

- p' = the horizontal pressure produced by a line load per unit of wall length;
- $\overline{p}$ = the effective unit pressure;
- p_w = pore water pressure;
- p_1 = the horizontal unit pressure produced by a point load along the intersection between the inner face of a wall and a vertical section through the point load, perpendicular to the wall;

Q = a point load;
q = a uniformly distributed surcharge, per unit of area:

q' = a unit line load;
q_u = the unconfined compressive strength of cohesive soil;
s = the shearing resistance, per unit of area;
w = the natural water content, in percentage of the dry weight;
$x, y, z,$ = the variable distances in the directions X, Y, and Z;
α = the ratio between the depth of the dredge line below the surface of the backfill and the length of the sheet piles;
β = the ratio between the depth of the anchor line below the surface of the backfill and the length of the sheet piles;
γ = the unit weight of a soil including the weight of water contained in its voids:
$\bar{\gamma}$ = the effective unit weight of silt or clay (the saturated unit weight above the water table and the submerged unit weight below the water table);
γ' = the submerged unit weight;
$\Delta\gamma'$ = the reduction in the submerged unit weight resulting from seepage pressure exerted by rising ground water;
$\Delta\gamma''$ = the increase in the effective unit weight;
γ_w = the unit weight of water;
δ = the angle of wall friction;
μ = the Poisson ratio;
ρ = the flexibility number;
ρ_a = the flexibility number corresponding to the point of intersection N in Fig. 16.
ρ_c = the critical flexibility number at which M becomes smaller than M(max);
σ = the normal stress;
σ_x = the horizontal unit pressure;
ϕ = the angle of internal friction;
ϕ' = the angle of partly mobilized internal friction; and
ψ = an angle defined by Fig. 8(*b*).

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